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Upgrading Existing Wastewater Treatment Plants Case Histories

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Upgrading Existing Wastewater Treatment Plants

Case Histories

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UPGRADING EXISTING WASTEWATER TREATMENT PLANTS- CASE HISTORIES

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Part I

UPGRADING THROUGH BIOLOGICAL—PROCESS MODIFICATION

The case histories herein relate to upgrading the performance of activated-sludge and trickling-filter plants through modification to the biological processes.

In these processes, the biochemical reaction rates depend on the concentration of reactants (wastes) and of the appropriate micro-organisms capable of feeding on the wastes.

For high degrees of treatment, rates of biochemical-oxygen-demand (BOD) removal per unit weight of micro-organism drop off. Equilibrium is reached when the rate of BOD removal is balanced by BOD from lysis products of dead micro-organisms. Equilibrium-soluble BOD levels vary from system to system, but typically are above 5 mg/l. For biological systems that include effluent filtration or other effective means of removing fine solids from the effluent, the minimum equilibrium-soluble BOD level provides the target for optimization.

In actual systems the following factors may limit performance to less than optimum.

INADEQUATE HYDRAULIC-RETENTION OR CONTACT TIME

This condition is generally a significant limiting factor only where

- High removals are required (effluent BOD < 20 ppm).
- The system operates on short contact times, e.g., trickling filter or contact stabilization (also for a fraction of flow in completely mixed or step-aeration systems).
- Wastes contain constituents not readily adsorbed on microbial flocs or slimes.

Most BOD constitutents of domestic sewage are adsorbed rapidly, provided the micro-organisms in the system have had adequate opportunity to assimilate previously adsorbed material.

Some BOD constitutents—particularly industrial wastes with high dissolved fractions—do not show rapid initial removal. Whether this is an inherent waste characteristic or is related to the population characteristics of the micro-organisms generally has not been established for particular systems.

SOLIDS RETENTION TIME

This parameter determines the characteristics of the microbial population in a treatment system; types of organisms present, reaction rates for particular waste constituents, settling characteristics, and so forth.

Solids retention time (SRT) reflects

- The biological loading on the system: overall, pounds BOD per day per pound mixed-liquor volatile suspended solids (MLVSS)
- The cell yield for the particular waste constituents and biological solids in the system: overall, pounds solids per pound BOD removed
- The rate of auto-oxidation of the biological solids

In general, systems with high loadings show low SRT's.

Increasing SRT will, in general, improve performance. In lightly loaded systems, however, increase in solids retention may actually reduce performance by

- Increasing opportunities for return of BOD to system from lysis of biological solids
- Reducing settleability of biological solids (especially in activated-sludge systems)

Hence, to optimize performance, an operator needs not only treatment facilities of adequate capacity but also provisions for control of the solids in the system (variable return-sludge and waste-sludge rates in activated sludge, variable recirculation in trickling filters, etc.).

Most systems in need of upgrading, however, suffer from inadequate SRT's—especially where a particular waste component is removed only by relatively slow-growing micro-organisms. If the SRT in the system is less than the generation time of the particular micro-organisms, the micro-organisms will be washed or crowded out of the system. In this case the particular waste component will not be removed or oxidized at all. The most important example of this effect is the oxidation of ammonia in activated-sludge or trickling-filter systems. Typically, no nitrification results in single-stage systems unless loadings are low enough to permit long SRT's.

Solids retention times may be increased by

- Increasing reactor volume
- Increasing the concentration of biological material in the reactor

Generally, cost is the only limit on increasing reactor volume. Increasing reactor solids levels may involve no direct costs, but may be limited by the other related factors.

In activated-sludge systems these factors include:

- Higher solids loadings imposed on final clarifiers
- Ability to supply oxygen to the reactor and to the interior of sludge floc particles themselves

In trickling filters or other fixed-film reactors these factors include:

- The maximum media surface per unit volume of reactor that can be employed without clogging (high area per unit volume requires fine media)
- Ventilation of reactor and ability to diffuse oxygen and waste constituents into biological films

In increasing the performance of a particular plant the foregoing factors must be examined, and all those limiting performance at the desired level must be modified. Table I-1 indicates specific process modifications that should be considered to overcome particular limitations on system performance.

Table I-1.—*Process modifications*

Modification or addition	When to use	Remarks
Tighten process control	To minimize soluble BOD in aerator effluent and improve settling characteristics of sludge	Requires flexibility in return, waste sludge and air rates, and requires control instrumentation.
Add final tank capacity	If solids loading on final tanks limits the solids level that can be maintained in aerator	Check buildup of sludge in clarifiers at high flow rates.
Increase aerator solids level	To increase removals of waste constituents assimilated by slow-growing micro-organisms	Consider effect on final tanks, and adequacy of oxygen supply.
Increase air supply or improve its distribution	Where DO cannot be maintained above 1-3 ppm at all loads and locations	Consider oxygen aeration.
Increase contact time	In contact stabilization system where tests indicate rapid initial removal is below level desired	Check effects on settling characteristics of sludge.
Divide reactor into multiple compartments in series	Where short circuiting has significant effect on BOD levels in aerator effluent	
Contact stabilization	For upgrading overloaded plug flow or multicompartmented tanks where initial removal is adequate	Avoid in small plants with wide diurnal flow variations.
Step aeration	For upgrading overloaded plug flow or multicompartmented tanks to higher removals than possible with contact stabilization	
Completely mixed activated sludge	For upgrading any conventional activated-sludge system and for conversion of small contact-stabilization plants	Piping changes will be very costly for long plug-flow tanks; step aeration preferable in such cases.

Table I-1.—*Process modifications—Concluded*

Trickling filters		
Modification or addition	When to use	Remarks
Positive ventilation	For strong wastes and deep filters where natural draft is not adequate to provide oxygen needed	Check with media manufacturers.
Media change	In rock filters where increase in media surface (and biomass) will provide needed performance improvement	Synthetic media are available with from 2-4 times the surface-to-volume ratio of stone. Fine media cannot be used for high loadings due to clogging.
Increase recirculation	For lightly loaded filters where initial removal capacity is not fully utilized in one-pass contact time	May also be needed for dilution to maintain aerobic conditions in heavily loaded filters.
Activation (sludge recirculation)	Where more biomass is needed to increase treatment	Requires thorough pilot study.[1]
High rate trickling filter (roughing)	Ahead of overloaded trickling-filter or activated-sludge units	Intermediate settling not needed. Performance quite temperature sensitive.
Activated sludge		May be designed for nitrification. Requires extra set of settling tanks.
	After trickling-filter or activated-sludge plant	Advantageous when first unit can be converted to "high rate" (high loading) operation.
	Ahead of trickling filter	Limit to special cases such as industrial pretreatment.
	Ahead of activated sludge	Use where existing facilities are suitable for final stage.
Nitrification trickling filter	After settling tanks of existing secondary system	Provides polishing and nitrification. Because of low sludge yield, effluent probably can be applied directly to granular-media filters for solids removal. Advantageous where effluent filters are to be included in upgrading.

[1]Refer to *Water and Wastewater Engineering*, 34, Aug. 1970.

Part II

CASE HISTORIES

CASE 1. GREENSBORO, N.C., SOUTH BUFFALO CREEK WASTEWATER TREATMENT PLANT

The original South Buffalo Creek Wastewater Treatment Plant was built in 1931 for an ultimate population of 37,000, an average flow of 3.25 mgd, and a wet-weather flow of 6.5 mgd. The plant was adequate generally for the design load, except that the final settling tank, sludge-digestion tank, and sludge-drying beds were considerably smaller than would be called for by modern practice. These deficiencies resulted in somewhat less satisfactory treatment than ordinarily would be expected with this type of plant.

Facilities in the original plant consisted of screens, detritors, single primary and final settling tanks, two fixed-nozzle trickling filters, a digester, and sludge-drying beds.

By 1956 the flow to the plant had increased to an average of 4 mgd, of which 1.5 mgd were industrial waste. The total load to the plant was equivalent to that from 52,000 people, and the treatment efficiency was inadequate. Among the industries contributing to the plant were textile-dyeing and -finishing plants, an abattoir, a meatpacking plant, a chemical-manufacturing plant, poultry-processing plants, and metal-plating plants.

The treatment plant was upgraded in 1957-58 by replacement of screening and grit-removal facilities; addition of one primary settling tank, two trickling filters, two final tanks, and one digester; conversion of the existing final tank to chlorine contact; and replacement of the drying beds with a vacuum-filter installation.

Between 1958 and 1964, the industrial BOD contribution to the plant increased much more rapidly than expected. Industrial BOD rose from 35 percent to over 65 percent, and the overall plant BOD-removal efficiency fell from 90 percent to 70 percent.

To improve handling of the industrial loads one of the original fixed-nozzle trickling filters was converted into two aeration basins in 1964-66. Conversion involved removal of media and installation of platform-mounted mechanical aerators. The trickling filters installed in 1957 were retained as roughing filters ahead of the activated-sludge facilities.

Since the initial conversion, BOD removals have averaged above 90 percent, and have never dropped below 80 percent.

In 1970, to meet increased loads, the remaining fixed-nozzle filter was converted to aeration basins using floating aerators.

The current upgrading project includes preaeration, chemical addition for phosphorus removal, special odor-control measures, improved sludge handling, and effluent polishing with deep-bed filters. BOD and suspended-solids removals in excess of 98 percent are expected.

Design parameters of the various plant additions are detailed in table II-1, typical plant-operating data are given in table II-2, and capital costs of plant upgrading are outlined in table II-3. Flow diagrams for various stages in development of the plant are shown in figures II-1 to II-5.

Included is a possible future upgrading to provide nitrogen removal, should this be required. This upgrading might involve conversion of the 1957 trickling filters to provide nitrification after the activated-sludge treatment and addition of biological denitrification.

Table II-1—*Upgrading of South Buffalo Creek Wastewater Treatment Plant, design parameters*

Item	1931 (original plant design)	1957	1964-66	1970	1972 proposals
Design parameters:					
Design average flow, mgd	3.25	8	8	8	10.7
Design peak flow, mgd	6.50	16	16	16	21.4
Design BOD, lb/day	–	21,700	–	–	35,500
Design suspended solids, lb/day	–	18,800	–	–	27,000
Phosphorus, lb/day	–	–	–	–	900
Wastewater treatment:					
Screens	Mechanical bar screen.	Existing bar screen replaced by back-raked mechanical bar screen with hand-raked bypass screen.	No change.	No change.	Replace mechanical bar screen by new heavy-duty unit.
Grit chambers	Two detritors.	Detritors replaced by aerated grit chamber. Available air supply: 50 ft^3/min. Grit removal: tubular conveyor discharging to grit box.	No change.	No change.	Replace grit-removal mechanism with new heavy-duty bucket-type grit-collecting-and-elevating mechanism.
Preaeration tank	–	–	–	–	Volume: 1.1 million gallons. Available air supply: 4,000 ft^3/min.
Primary settling tanks	Square tank. Dimensions: 70 ft x 70 ft x 10 ft SWD. Surface area: 4,900 ft^2. Overflow rate: 670 gpd/ft^2.	Old square tank retained. New round tank (80 ft diameter x 9 ft SWD) added. Flow equally divided between tanks. Total surface area: 9,900 ft^2. Overflow rate: 800 gpd/ft^2.	No change.	No change.	Enclose primary tanks with positively ventilated building to exhaust air through odor-control units.
Phosphorus removal	–	–	–	–	Addition of facilities to add lime, alum, or iron salts (depending on results of pilot-plant studies) ahead of primary settling tanks.

Table II-1.—*Upgrading of South Buffalo Creek Wastewater Treatment Plant, design parameters—Continued*

Item	1931 (original plant design)	1957	1964-66	1970	1972 proposals
Secondary treatment	Two square (200 ft x 192 ft x 7 ft deep) fixed-nozzle, stone trickling filters. Media volume: 540,000 ft^3.	Square filters retained, accepting flow from old ordinary tank. Loading: 15-lb BOD/ 1,000 ft^3 of media per day. Two new round filters (155 ft diameter x 7 ft deep) added to accept flow from new primary tank. Media volume 260,000 ft^3 loading 36 lb BOD/ 1,000 ft^3 of media per day. New filters have recirculation capacity of 9 mgd.	Use of square filters discontinued. Round filters operated as roughing filters. One square filter converted to two aeration basins with combined volume of 326,400 ft^3 and combined aeration capacity of 200 hp of platform-mounted aerators.	Second square filter converted to provide additional two aeration basins with 360 hp aeration capacity, giving total aeration volume of 655,000 ft^3 and total aeration capacity of 560 hp.	No change.
Final settling	One rectangular tank (50 ft x 50 ft x 10 ft SWD). Total surface area: 2,500 ft^2. Overflow rate: 1,300 gpd/ft^2.	Old final settling tank converted to chlorination contact tank (see below). Two new round (105 ft diameter x 9 ft SWD) tanks added to handle total plant flow. Total surface area: 17,300 ft^2. Overflow rate: 480 gpd/ft^2.	No change.	No change.	No change.
Chlorination	None.	Contact tank volume: 25,000 ft^3 Chlorinator capacity: 4,000 lb/day.	No change.	No change.	No change.
Effluent filtration .	–	–	–	–	Preaeration followed by dual media filtration (design dependent on results of pilot-plant studies).
Residue treatment:					
Sludge thickening .	–	–	–	–	Flotation thickening of activated sludge prior to mixing with primary-sludge flotation unit. Surface area: 250 ft^2.
Sludge digestion . .	Fixed-cover digester 60 ft diameter x 23 ft SWD.	New floating-cover digester (80 ft diameter x 30 ft SWD) added as primary digester. Existing fixed-cover digester retained as secondary unit.	Digestion discontinued. Digesters retained as sludge-holding tanks.	No change.	No change.

Table II-1.—*Upgrading of South Buffalo Creek Wastewater Treatment Plant, design parameters—Concluded*

Item	1931 (original plant design)	1957	1964-66	1970	1972 proposals
Sludge dewatering .	Sludge-drying beds, area: 32,600 ft^2.	New 11 ft-6-in. x 12 ft large coil spring vacuum filter (430 ft^2 surface area) added. Drying beds relegated to standby role.	No change.	No change.	Recondition existing vacuum filter and accessories. Replace filter media. Provide odor control for sludge-handling building
Ultimate disposal . .	Trucked to landfill.	Trucked to landfill.	Trucked to landfill.	Trucked to incinerator.	No change.

Table II-2.—*Typical performance of South Buffalo Creek Wastewater Treatment Plant*

Item	1957-58		1964-66		1970		Proposed
	Before upgrading	After upgrading	Before upgrading	After upgrading	Before upgrading	After upgrading	After upgrading
Characteristics of raw waste:							
Flow:							
Average day, mgd	3.7	4.5	4.9	6.6	6.8	8.9	10.7
Peak hour, mgd	10	11.5	11.5	16	16	18	21
BOD:							
Average day, mg/l	390	310	290	300	390	390	400
Average day, lb/day	11,700	11,800	11,800	16,500	22,200	29,000	35,700
Suspended solids:							
Average day, mg/l	350	240	250	260	290	300	300
Average day, lb/day	10,800	9,100	10,200	14,100	16,500	22,300	26,800
Performance of treatment units:							
Average BOD removal, lb/day:[1] . .							
Primary	2,300(20)	3,000(25)	3,000(25)	2,500(15)	3,200(15)	4,400(15)	9,000(25)
Trickling filter	4,700(50)	7,400(85)	5,200(60)	5,800(40)	6,600(35)	7,600(30)	8,000(30)
Activated sludge	-	-	-	6,500(80)	9,000(75)	14,100(85)	16,000(85)
Effluent filter	-	-	-	-	-	-	2,000(75)
Total.	7,000(80)	10,400(90)	8,200(70)	14,800(90)	18,800(85)	26,100(90)	35,000(98)
Average SS removal, lb/day:[1]							
Primary	4,300(40)	4,800(50)	5,100(50)	4,200(30)	5,000(30)	7,800(35)	13,400(50)
Secondary.	2,200(35)	3,200(70)	2,500(50)	7,100(70)	7,400(65)	8,900(60)	10,500(80)
Effluent filter	-	-	-	-	-	-	2,300(80)
Total	6,500(60)	7,700(85)	7,600(75)	11,300(80)	12,400(75)	16,700(75)	26,200(98)
BOD loading on units (1,000 lb per ft^2):							
Trickling filter	17	[2]15/36	[2]15/36	54	73	95	103
Activated sludge	-	-	-	25	38	26	29
Operating costs, dollars per million gallons	-	-	38.50	52.93	58.91	78.28	-

[1]Figures in parentheses indicate percent.

[2]Two-stage; see table II-1.

Table II-3.—*South Buffalo Creek Wastewater Treatment Plant, capital costs of upgrading*

Date of upgrading	Item	Capital cost	
		Cost at time of upgrading	1972 cost
		Dollars	
1957	Screening and degritting	45,000	110,000
	Rehabilitation of pump and control building	150,000	365,000
	Primary settling	65,000	160,000
	Secondary process	330,000	800,000
	Final settling	135,000	330,000
	Chlorination facilities	25,000	60,000
	Sludge handling	300,000	730,000
	Miscellaneous	150,000	365,000
	Total, 1957	1,200,000	2,920,000
1964-66	Modifications to secondary process	200,000	367,000
1970	Modifications to secondary process	100,000	133,000
1972 proposal	Renovation of screens and degritting equipment	-	60,000
	Preaeration and odor control	-	160,000
	Enclosure of primary tanks and odor control	-	200,000
	Phosphorous-removal facilities	-	75,000
	Effluent filters and aerators	-	450,000
	Sludge handling	-	250,000
	Miscellaneous	-	55,000
	Total, 1972 proposal	-	[1]1,250,000

[1]Exclusive of engineering and contingencies.

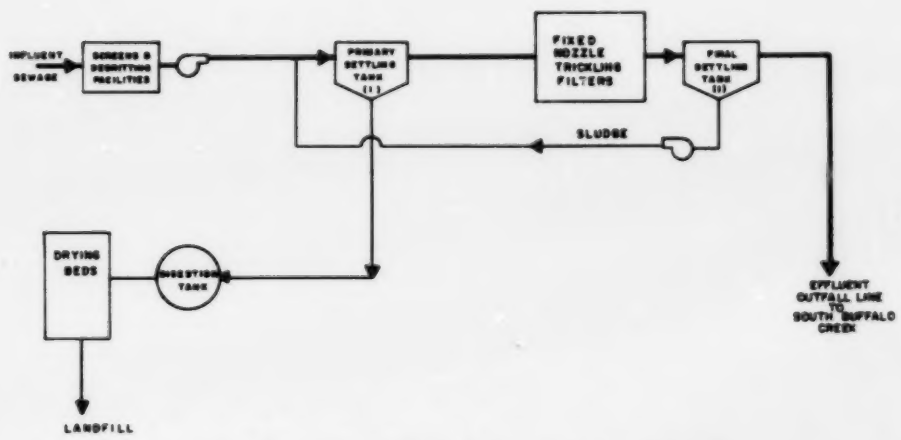

Figure II-1. South Buffalo original plant, flow diagram.

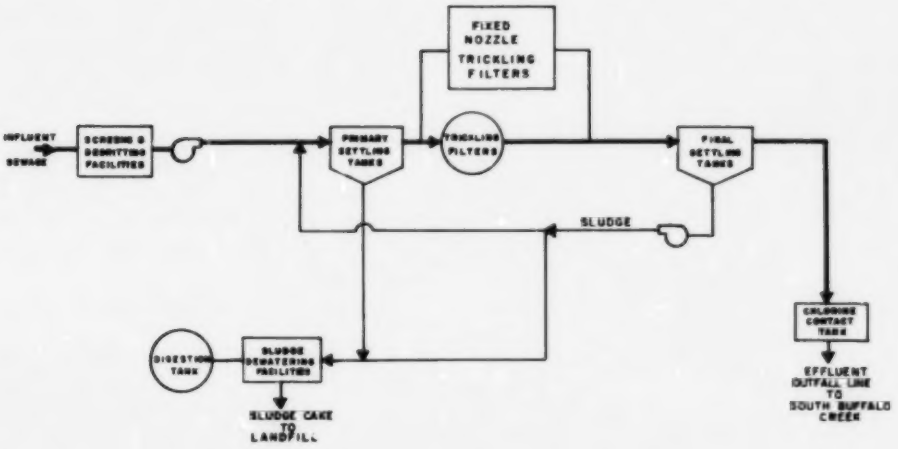

Figure II-2. South Buffalo, following first upgrade, flow diagram.

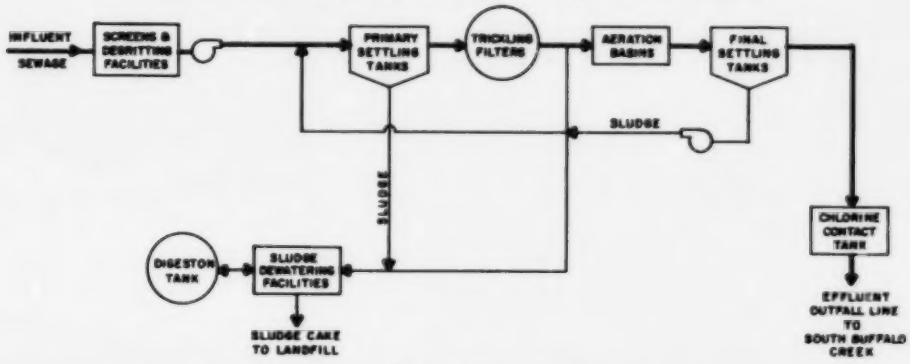

Figure II-3. South Buffalo, following second upgrade, flow diagram.

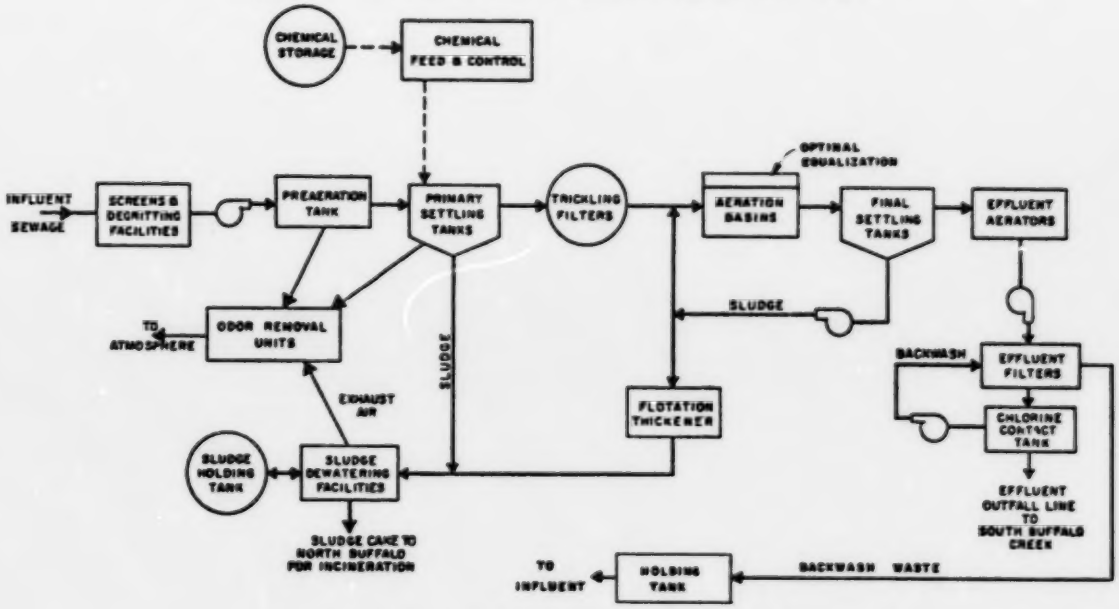

Figure II-4. South Buffalo, current upgrade, flow diagram.

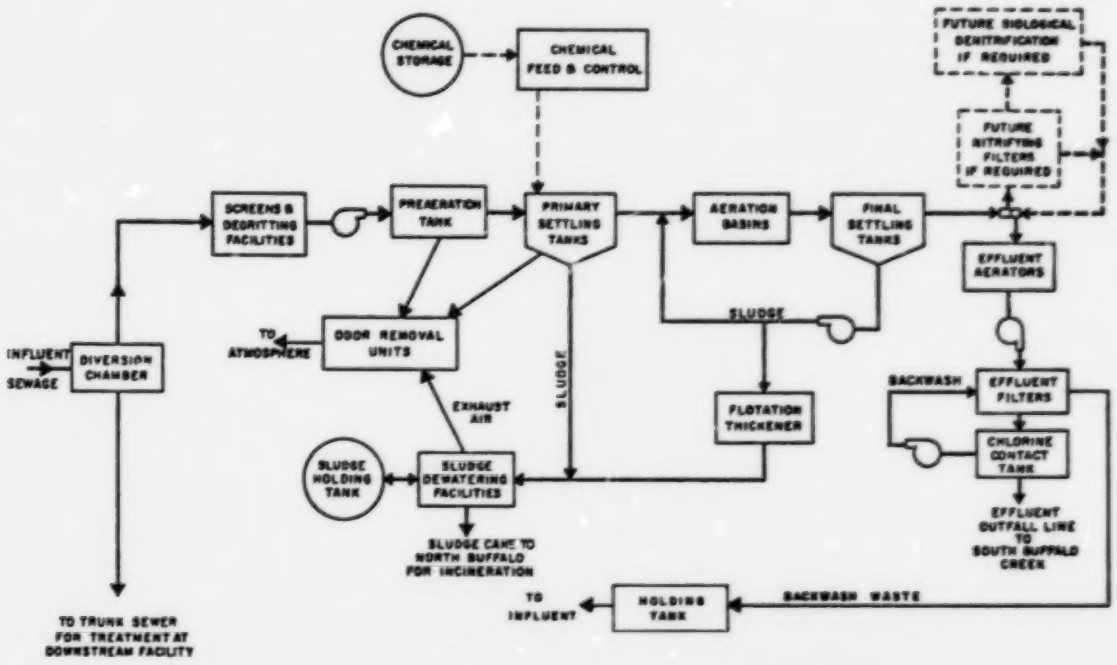

Figure II-5. South Buffalo, future upgrade, flow diagram.

CASE 2. LIVERMORE, CALIF., WASTEWATER TREATMENT PLANT

The Livermore Wastewater Treatment Plant was built in 1958 to provide secondary treatment for a domestic wastewater with an average dry weather flow of 2.5 mgd. The plant included preliminary treatment and roughing filters followed by 30-day oxidation ponds. Sludge was digested, then dewatered in lagoons. Effluent was discharged to an intermittently flowing drainage ditch. See figure II-6 for plant flow diagram and table II-4 for plant design data. Design effluent limitations were 40 mg/l BOD_5 and 40 mg/l suspended solids. The plant achieved effluents with 45-50 mg/l BOD_5 and 45-50 mg/l suspended solids, with no significant nitrification.

The plant was enlarged in 1967 to provide an increased degree of treatment for an average dry weather flow of 5 mgd. All existing structures were incorporated in the upgraded plant. The new plant includes preliminary treatment, roughing filters, activated-sludge secondary treatment, and pre- and postchlorination. The former oxidation ponds have been converted to the emergency holding ponds. The existing sludge-disposal system was expanded. Part of the effluent (about 20 percent) is used to irrigate a municipal golf course, an airport, and adjacent farmland. See figure II-7 for plant flow diagram and table II-5 for plant design data. Effluent limitations (Regional Water Quality Control Standards) were 20 mg/l BOD_5, 20 mg/l suspended solids, 1 mg/l grease, 5 MPN per 100 ml coliforms (using 5-day median). The plant easily meets these standards (see table II-6 for operating data). Complete nitrification must be provided to achieve the low effluent bacterial count (if nitrification is not complete, chlorine demand is inordinate because of chloramine formation, and inadequate free chlorine remains to effect the requisite high levels of disinfection).

The initial plant had a capital cost of $900,000 (1957 dollars), and an annual operating cost of $102,000 (1968 dollars). Capital cost of the expansion was $1,300,000 (1968 dollars). Annual operating cost is $270,000 (1972 dollars). Unit capital costs of upgrading are $520,000 per mgd of increased capacity, or $186 per pound per day of additional BOD_5 removal.

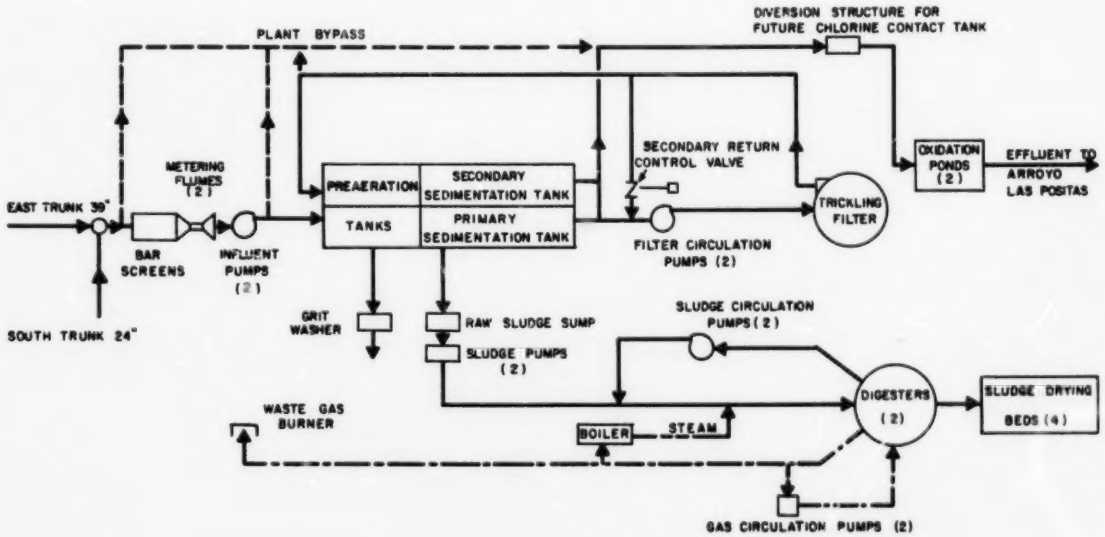

Figure II-6. Livermore Wastewater Treatment Plant, flow diagram, 1958.

Table II-4.—*Livermore Wastewater Treatment Plant design data, 1958*

Design flow, mgd:	
Average dry weather	2.5
Maximum dry weather	4.5
Peak storm rate	10
Design loadings:	
Population equivalent	28,900
Suspended solids, lb/day:	
Per capita	0.15
Design total	4,500
BOD, lb/day:	
Per capita	0.20
Design total	6,500
Preaeration tanks:	
Number[1]	2
Width, feet	19
Length, feet	38
Average water depth, feet	11.7
Detention time, hours (one tank)	0.6
Air supplied per tank, ft^3/min^2	200
Air supplied, ft^3/gal	0.12
Maximum hydraulic capacity, mgd	10

Table II-4.—*Livermore Wastewater Treatment Plant design data, 1958—Concluded*

Primary sedimentation tank:	
Width, feet	19
Length, feet	124
Average water depth, feet	9
Effluent weir length per tank, feet	164
Detention time, hours	1.5
Mean forward velocity, ft/min	1.4
Overflow rate, gal/ft^2/day at average dry weather flow	1,050
Maximum hydraulic capacity of tank, mgd	10
Assumed removal, percent:	
Suspended solids	60
BOD	35
Secondary sedimentation tank[3]	([3])
Trickling filter:	
Inside diameter, feet	110
Average depth of filter media, feet	4.25
Size of filter media, inches	2-4
Net area of filter surface, acres	0.218
Volume, acre-ft	0.92
Circulation ratio to average design dry weather flow	1.5-3
Loading:	
Rate per filter, mgd	7.5
Rate per surface acre, mgd	34.5
BOD, lb/day/acre-ft	4,600
Assumed removal, percent, filter plus secondary sedimentation:	
Suspended solids	60
BOD	75
Oxidation ponds:	
Total area, acres	37
Average water depth, feet	6
Detention time, days	30
BOD loading, lb/acre/day	28

[1]One preaeration tank used to aerate trickling-filter effluent prior to secondary sedimentation.
[2]200 ft^3/min supplied to primary aeration tank and 100 ft^3/min supplied to secondary aeration tank.
[3]Details as for primary, except removals.

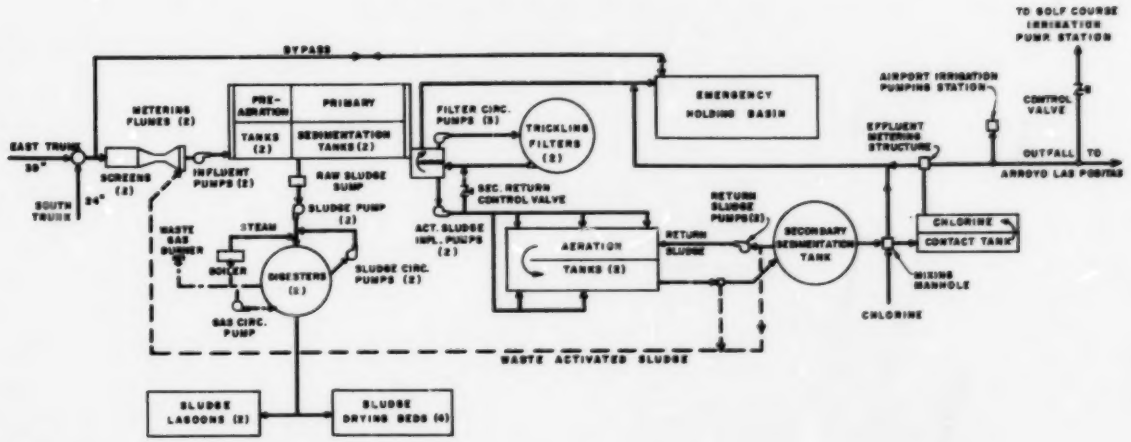

Figure II-7. Livermore Wastewater Treatment Plant, flow diagram, 1968.

Table II-5.—*Livermore Wastewater Treatment Plant, design data, 1968*

Design flow, mgd:	
Average dry weather	5
Maximum dry weather	10
Peak storm rate	18
Design loadings:	
Population equivalent	62,500
Suspended solids, lb/day:	
Per capita	0.20
Total	12,500
BOD, lb/day:	
Per capita	0.20
Total	12,500
Preaeration tanks:	
Number	2
Width, feet	19
Length, feet	38
Average water depth, feet	11.7
Detention time, hours	0.6
Air supplied per tank, ft^3/min	150
Hydraulic capacity, mgd	10
Primary sedimentation tanks:	
Number	2
Width, feet	19
Length, feet	124
Average water depth, feet	9
Effluent weir length per tank, feet	164
Detention time, hours	1.5
Mean forward velocity, ft/min	1.4
Overflow rate, gal/ft^2/day at average dry weather flow	1,050
Hydraulic capacity, mgd	10

Table II-5.—*Livermore Wastewater Treatment Plant, design data—Continued*

Primary treatment:	
Assumed suspended-solids reduction, percent	60
Suspended-solids reduction, lb/day	7,500
Assumed BOD reduction, percent	35
BOD reduction, lb/day	4,400
Trickling filters:	
Number	2
Inside diameter, feet	110
Average depth, filter media, feet	4.25
Size of filter media, inches	2-4
Net area of filter surface, acres	0.218
Volume, acre-ft per filter	0.92
Circulation ratio to average dry weather flow	1.5-3
Loading:	
Rate per filter, mgd	7.5
Rate per surface acre, mgd	34.5
BOD, lb/day/acre-ft	4,400
Assumed BOD removal, percent	50
BOD reduction, lb/day	4,000
Activated sludge aeration tanks:	
Number	2
Average water depth, feet	15
Width, feet	30
Length, feet	160
Detention time, hours, based on raw sewage flow	5.2
Air supplied, ft^3/lb BOD removed, ft^2/min	1,200
Volume per tank, ft^3	72,000
Volumetric loading, lb BOD per 1,000 ft^3	28
Return sludge, percent	10-100
Secondary sedimentation tanks:	
Number	1
Diameter, feet	90
Side water depth, feet	12
Detention time, hours, based on average dry weather flow	2.75
Overflow rate, gal per ft^2 per day at average dry weather flow	787
Activated-sludge treatment:	
Assumed suspended solids reduction, percent	88
Suspended solids reduction, lb/day	4,400
Assumed BOD reduction, percent	86
BOD reduction, lb/day	3,500
Overall plant performance:	
Assumed suspended solids reduction, percent	96
Assumed BOD reduction, percent	96
Effluent suspended solids, mg/l	15
Effluent BOD, mg/l	15

Table II-5.—*Livermore Wastewater Treatment Plant, design data, 1968—Concluded*

Chlorine contact tank:	
Volume, ft^3	28,000
Detention time at average dry weather flow, hours	1

Table II-6.—*Livermore Wastewater Treatment Plant, operating data, 1971*[a]

Item	Raw influent	Primary effluent	Trickling-filter effluent	Secondary effluent	Plant effluent
BOD, mg/l:					
Mean	213	127	93	9.9	7.3
High	244	156	129	16.9	12.4
Low	167	106	46	5.2	3.1
Suspended solids, mg/l:					
Mean	229	79	110	22	13
High	261	98	137	31	19
Low	187	60	74	15	8
Ammonia N, mg/l:					
Mean	-	40.7	32.6	1.68	.14
High	-	51.1	44.4	6.73	1.3
Low	-	33.8	22.5	.46	<.1
Nitrate N, mg/l:					
Mean	-	-	-	18.5	21.5
High	-	-	-	20.2	30
Low	-	-	-	16.6	14
Grease, mg/l:					
Mean	-	-	-	-	.23
High	-	-	-	-	.46
Low	-	-	-	-	.15
Coliforms, MPN per 100 ml:					
Mean	-	-	-	-	2.5
High	-	-	-	-	[1]7
Low	-	-	-	-	.4

[a]Monthly average. If value not given, item not tested for.

[1]When coliform count exceeded 5 MPN per 100 ml, flow diverted to emergency holding pond. Discharge not resumed until count of less than 5 achieved.

CASE 3. NEW YORK, N. Y., WARDS ISLAND WASTEWATER TREATMENT PLANT

The Wards Island Wastewater Treatment Plant was built in 1937 to provide secondary treatment for an average flow of 180 mgd. By 1947 the average flow was in excess of 226 mgd; solids-removal

performance had become erratic and air consumption was excessive. These problems were remedied and the capacity was increased to 240 mgd by two inexpensive plant modifications.

- The conventional activated-sludge aerators were converted to operate on the step-aeration process, which was conceived by Gould for New York City treatment plants. This was the first full-plant installation.

- The flow pattern in the final tanks was changed.

The original design provided 16 aeration tanks, with sufficient aeration volume for 5.25 hours detention at 125 percent of average flow, with one tank out of service. The modification of the tank influent systems to provide step aeration (see fig. II-8) cut the requisite aeration detention time to 2.5 hours, with no deleterious effect on performance and greatly enhanced operational flexibility. Using the step-aeration process, 12 of the original aerators had sufficient capacity for over 240 mgd. A study of plant capacity carried out in 1965 concluded that, if the existing preliminary tanks were replaced and the four surplus aerators converted into final settling tanks, the plant could be rated for at least 250 mgd.

There are 32 final tanks at Wards Island, each 179 feet long by 425 feet wide; original design overflow rate was 1,000 gpd at 125 percent of average flow, with two tanks out of service. Before modification, aeration-tank effluent was fed from a center conduit and the final tank effluent passed over weirs at both ends. Sludge was withdrawn from a cross-collector hopper at the center of the tank. Tank flow was affected strongly by density currents, which carried solids away from the sludge drawoff at a rate approaching the solids-handling capacity of the sludge mechanism. This effect resulted in the formation of sludge blankets in excess of 2 feet, and the subsequent long sludge detention under anaerobic conditions increased oxygen demand in the aeration tank and adversely affected the settling qualities of the sludge, with resulting high solids concentrations in the final effluent. Alterations to the tank (see fig. II-9) consisted of abandoning center feed and providing greatly improved influent-effluent structures. Flow now passes the entire length of the tank. Under the modified flow pattern, density currents aid the mechanism in moving sludge to the drawoff more rapidly. Solids carryover has been eliminated, and the sludge blanket is consolidated to a depth of only a few inches above the depth of the collecting-mechanism flights.

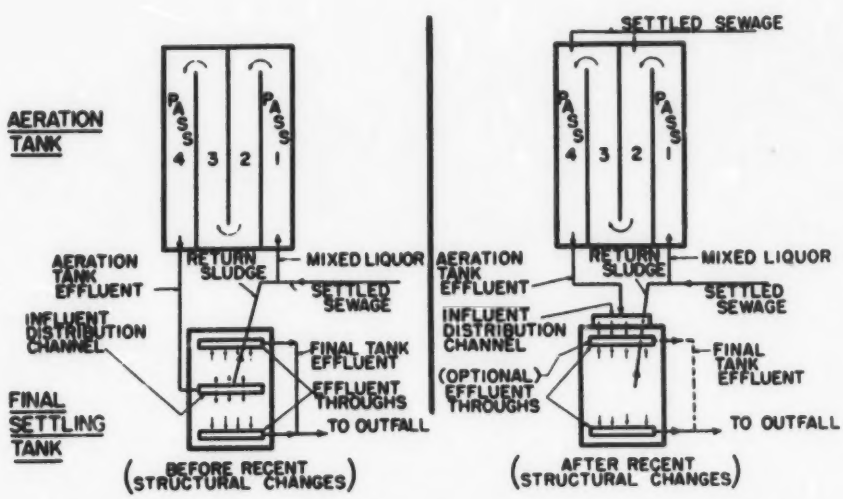

Figure II-8. Wards Island Wastewater Treatment Plant, aeration-tank modifications. Reproduced from Gould, *Sewage and Industrial Wastes,* Vol. 22, No. 8, p. 1000.

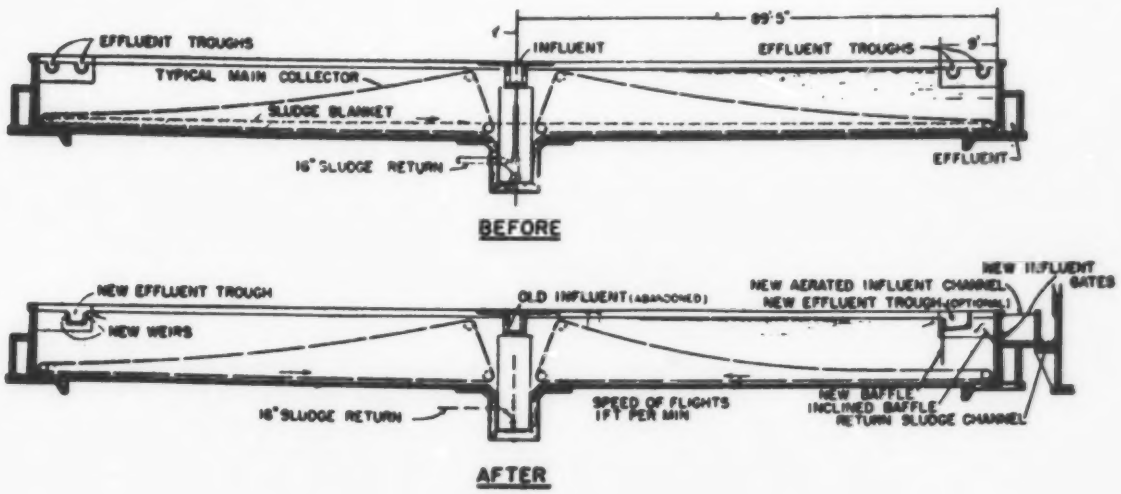

Figure II-9. Wards Island Wastewater Treatment Plant, final tank modifications. Reproduced from Gould, *Sewage and Industrial Wastes,* Vol. 22, No. 8, p. 1000.

The initial capital cost of the Wards Island Plant was $13,000,000 (1937 dollars). The alterations to the final tanks cost $260,000 (1949 dollars); aeration tank modifications cost $134,000 (1949 dollars). The cost of converting the surplus aerators to final tanks is estimated at $1,400,000 (1965 dollars). The unit capital cost of upgrading the secondary treatment capacity of the plant to 250 mgd would be $30,000 (1965 dollars) per million gallons per day of increased capacity.

CASE 4. UPGRADING EXISTING TRICKLING-FILTER PLANT BY ADDITION OF ACTIVATED SLUDGE AHEAD OF FILTERS

A treatment plant, constructed in 1964 at a cost of $1,500,000 (1964 dollars), was designed to give secondary treatment to an average flow of 4.8 mgd with a nominal BOD_5 load of 12,000 lb/day, including 8,500 lb/day of BOD_5 expected from a proposed food-processing plant. The plant included conventional primary treatment, trickling filters, final settling, and chlorination. Sludge was digested, then dewatered on sand drying beds. See figure II-10 for plant flow diagram.

By 1965, the total waste loading to the treatment plant proved to be greater than originally expected, and operating problems developed. Odors developed at preliminary tanks and at the trickling filters. The final settling-tank sludge was dilute, and would not settle well. Fine solids carried over from the final tanks to the effluent. The dilute final-tank sludge was pumped to the plant influent for resettling in the preliminary tanks; the resulting effluent then also contained fine solids. The mixed sludge from the preliminary tanks was also dilute, and the excess water caused reduced detention time and poor stabilization in the plant anaerobic digester.

Initial modifications made at the treatment plant included

- Dilution of raw wastewater with stream flow to increase its oxygen content and reduce its strength
- Covering of the preliminary tanks and trickling filter with ozonation of the exhaust gases for odor control
- Pumping of final tank sludge to a separate thickening tank, before repumping to the digesters

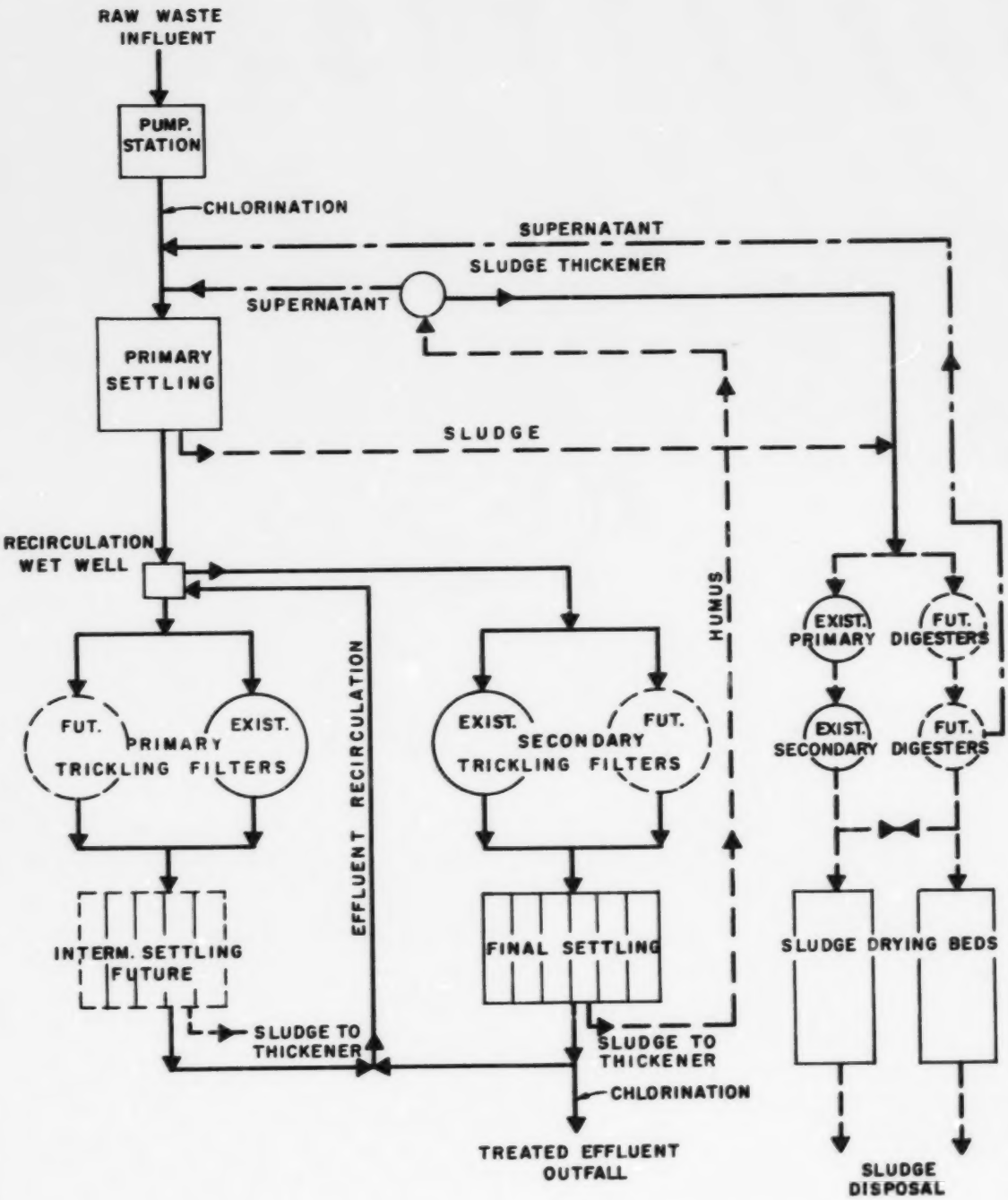

Figure II-10. Trickling-filter wastewater-treatment plant, flow diagram.

A survey of the processing-plant waste in early 1967 showed an average workday flow of 1.25 mgd containing 13,500 pounds of BOD_5, consisting chiefly of dissolved sugars and similar carbohydrates. Laboratory and pilot-plant tests indicated that the waste was amenable to treatment by activated sludge, if nutrients were added. It was decided that the most economical form of additional treatment capacity would utilize the activated-sludge process with lagoon-type aeration basins.

Because of lack of space at the treatment plant for such units, the basins were constructed upstream near the processing plant, with the effluent discharged to the sewer system.

The pretreatment plant was designed for a loading of 1.50 mgd with a BOD_5 of 15,000 lb/day, and to achieve a BOD_5 reduction of 75 percent. See figure II-11 for plant flow diagram. This plant went into operation in late 1968; it consistently provides good BOD_5 removal (80-90 percent) by maintaining a loading of 0.3-0.4 lb BOD_5 per pound mixed-liquor suspended solids (MLSS) at 15°-22° C. The reduced loading at the main treatment plant has permitted it to operate with a minimum of problems. The sludge from the activated-sludge units is filamentous, with a generally high sludge-volume index and poor settling characteristics. It is stabilized onsite in an aerated lagoon, and then dried on sand beds. This procedure has greatly relieved the overloaded sludge-handling facilities at the main plant.

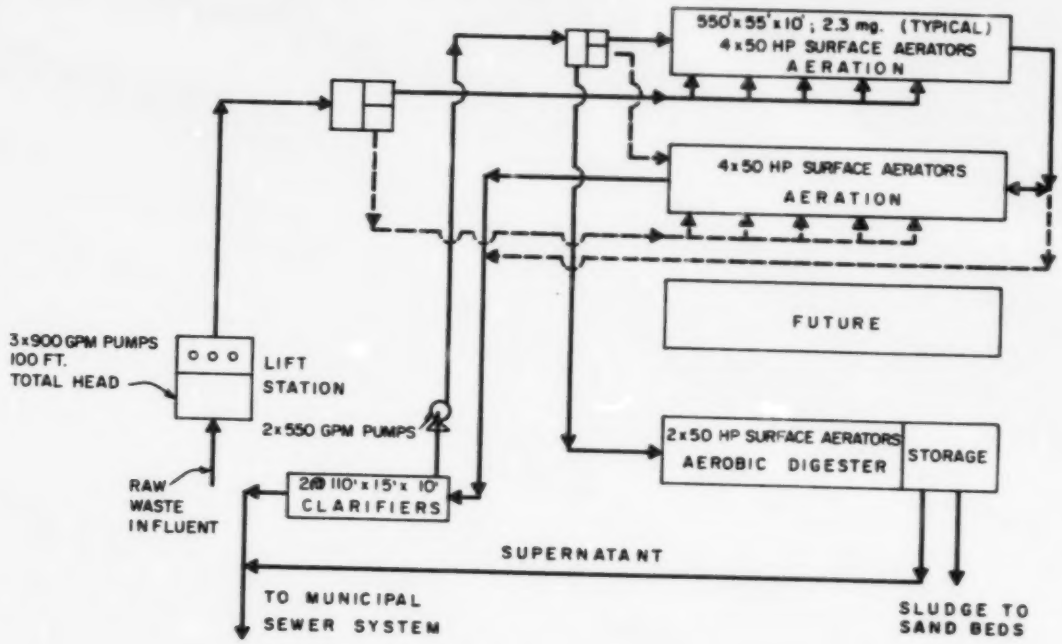

Figure II-11. Activated-sludge pretreatment ahead of trickling filters, flow diagram.

The additional treatment facility had a capital cost of $1,000,000 (1968 dollars); annual operating cost is $100,000 (1972 dollars). Unit capital cost of upgrading was $83.5 per pound BOD_5 removed (based on 80 percent BOD_5 reduction).

METRIC CONVERSION TABLES

Recommended Units

Description	Unit	Symbol	Comments	Customary Equivalents
Length	metre	m	*Basic SI unit*	39.37 in. = 3.28 ft = 1.09 yd
	kilometre	km		0.62 mi
	millimetre	mm		0.03937 in.
	micrometre	μm.		$3.937 \times 10^{-3} = 10^{-3}$A
Area	square metre	m^2		10.764 sq ft = 1.196 sq yd
	square kilometre	km^2		6.384 sq mi = 247 acres
	square millimetre	mm^2		0.00155 sq in.
	hectare	ha	The hectare (10 000 m^2) is a recognized multiple unit and will remain in international use.	2.471 acres
Volume	cubic metre	m^3		35.314 cu ft = 1.3079 cu yd
	litre	l	The litre is now recognized as the special name for the cubic decimetre.	1.057 qt = 0.264 gal = 0.81×10^{-4} acre-ft
Mass	kilogram	kg	*Basic SI unit*	2.205 lb
	gram	g		0.035 oz = 15.43 gr
	milligram	mg		0.01543 gr
	tonne or	t	1 tonne = 1 000 kg	0.984 ton (long) = 1.1023 ton (short)
	megagram	Mg	1 Mg = 1 000 kg	
Time	second	s	*Basic SI unit*	
	day	d	Neither the day nor the year is an SI unit but both are important.	
	year	year		
Force	newton	N	The newton is that force that produces an acceleration of 1 m/s^2 on a mass of 1 kg.	0.22481 lb (weight) = 7.233 poundals
Moment or torque	newton metre	N-m	The metre is measured perpendicular to the line of action of the force N. Not a joule.	0.7375 ft-lbf
Stress	pascal	Pa		0.02089 lbf/sq ft
	kilopascal	kPa		0.14465 lbf/sq in

Recommended Units

Description	Unit	Symbol	Comments	Customary Equivalents
Velocity linear	metre per second	m/s		3.28 fps
	millimetre per second	mm/s		0.00328 fps
	kilometres per second	km/s		2.230 mph
angular	radians per second	rad/s		
Flow (volumetric)	cubic metre per second	m^3/s	Commonly called the cumec	15,850 gpm = 2.120 cfm
	litre per second	l/s		15.85 gpm
Viscosity	pascal second	Pa·s		0.00672 poundals/sq ft
Pressure	newton per square metre or pascal	N/m^2		0.000145 lb/sq in
		Pa		
	kilometre per square metre or kilopascal	kN/m^2		0.145 lb/sq in.
		kPa		
	bar	bar		14.5 lb/sq in.
Temperature	Kelvin	K	*Basic SI unit*	$\frac{5F}{9} - 17.77$
	degree Celsius	C	The Kelvin and Celsius degrees are identical. The use of the Celsius scale is recommended as it is the former centigrade scale.	
Work, energy, quantity of heat	joule	J	1 joule = 1 N-m where metres are measured along the line of action of force N.	2.778×10^{-7} kw-hr = 3.725×10^{-7} hp-hr = 0.73756 ft-lb = 9.48×10^{-4} Btu
	kilojoule	kJ		2.778 kw-hr
Power	watt	W	1 watt = 1 J/s	
	kilowatt	kW		
	joule per second	J/s		

Application of Units

Description	Unit	Symbol	Comments	Customary Equivalents
Precipitation, run-off, evaporation	millimetre	mm	For meteorological purposes it may be convenient to measure precipitation in terms of mass/unit area (kg/m^2). 1 mm of rain = 1 kg/m^2	
River flow	cubic metre per second	m^3/s	Commonly called the cumec	35.314 cfs
Flow in pipes, conduits, channels, over weirs, pumping	cubic metre per second	m^3/s		
	litre per second	l/s		15.85 gpm
Discharges or abstractions, yields	cubic metre per day	m^3/d	1 l/s = 86.4 m^3/d	1.83 × 10^{-3} gpm
	cubic metre per year	m^3/year		
Usage of water	litre per person per day	l/person day		0.264 gcpd
Density	kilogram per cubic metre	kg/m^3	The density of water under standard conditions is 1 000 kg/m^3 or 1 000 g/l or 1 g/ml.	0.0624 lb/cu ft

Application of Units

Description	Unit	Symbol	Comments	Customary Equivalents
Concentration	milligram per litre	mg/l		1 ppm
BOD loading	kilogram per cubic metre per day	kg/m^3d		0.0624 lb/cu ft day
Hydraulic load per unit area; e.g. filtration rates	cubic metre per square metre per day	m^3/m^2d	If this is converted to a velocity, it should be expressed in mm/h (1 mm/s = 86.4 m^3/m^2 day).	3.28 cu ft/sq ft
Hydraulic load per unit volume; e.g., biological filters, lagoons	cubic metre per cubic metre per day	m^3/m^3d		
Air supply	cubic metre or litre of free air per second	m^3/s		
		l/s		
Pipes diameter	millimetre	mm		0.03937 in.
length	metre	m		39.37 in. = 3.28 ft
Optical units	lumen per square metre	lumen/m^2		0.092 ft candle/sq ft